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THE JOHN E. MATHEWS BRIDGE

by Maurice N. Quade and
Richard S. M. Lee

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THE JOHN E. MATHEWS BRIDGE
Jacksonville, Fla.

by

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Until it was named in honor of one of Jacksonville's leading citizens, the John E. Mathews Bridge was known as the Arlington Bridge. Arlington is located on the east bank of the St. Johns River and is much the smaller of the two communities which the bridge joins. The bridge is the first high-level bridge to be constructed in the State of Florida. A second high-level bridge having a somewhat longer main span is now under construction across Lower Tampa Bay on the West Coast.

The John E. Mathews Bridge is the largest of three major bridges included in the Jacksonville Expressway and financed by revenue bonds. It was opened to traffic on April 15, 1953 and is operated as a toll bridge. Together with portions of the Expressway system, it offers to both local and through traffic an effective and efficient by-pass of the traffic congestion in downtown Jacksonville as well as convenient and swift access to the beaches and the suburbs to the east and southeast of the City.

At the bridge site, the St. Johns River is almost exactly one mile wide between shores. The river is tidal and its shores are low and flat. The Jacksonville shore is occupied by industries and residences and has an average elevation of about 10 ft. above mean sea level. The Arlington shore is about 12 ft. higher and is residential.

The river has two channels - both dredged. The larger and more important one is called the Terminal Channel. The required minimum navigation clearances are 700 ft. horizontal by 150 ft. vertical. Unfortunately from the viewpoint of cost and aesthetics, the main channel is located near the west shore. The smaller channel is known as the Arlington Cut. Prior to dredging the Terminal Channel it was the only ship channel in the river at the bridge site. To the designers of this project it is still largely a "paper" channel since its exact location and the required clearances were rather fluid and elusive during the early stages of the work. However, no objections were raised to the adopted layout which provides minimum clearances of 362 ft. horizontal by 89 ft. vertical, and now that the bridge has been completed, the location of one point on the channel and the clearances are definitely known.

DESCRIPTION

The span over the Terminal Channel is a conventional through cantilever span, 810'-0" long c.c. piers. It is flanked on either side by a 405-ft. anchor span, but at that point, the symmetry ends. The west anchor span is close to the shore and the west approach consists entirely of deck plate girder spans. These spans vary in length from 113'-9" at the high end of the approach to 78'-9" at the low end. Two intermediate lengths of 100'-0" and 94'-0" were used. It was possible to provide crossings over three streets in Jacksonville and a single freight track without disturbing the uniform gradation of the span lengths. The total length of the west approach is 2307'-3".

East of the east anchor pier, the crossing is entirely over water except for a short length over an artificial island formed by spoil from channel dredging. The island proved to be a nuisance - except for surveys - since the contractor had to shift to land equipment to build the six piers located on it. The amount of excavation was increased because the pier bases had to be placed at substantially the same depths as the adjacent piers in the water in order to insure lateral stability.

Three spans in the east approach adjacent to the east anchor span are also through trusses. The first is a 270-ft. simple span. This is followed by a 270-ft. anchor span and a 440-ft. cantilever span over the Arlington Cut having only one cantilever arm 101 ft. long supporting one end of a 330-ft. suspended span. The opposite end of the suspended span rests on Pier 5E.

Continuing downgrade from Pier 5E, there are 14 deck girder spans varying from 113'-9" to 78'-9" in length. These spans are identical with those in the west approach.

It was originally intended to terminate the structure 800 ft. west of the east bank of the river and to utilize hydraulic embankment in shallow water for the remainder of the crossing. However, the riparian rights necessary for the embankment slopes were not obtained and it became necessary to extend the structure using rolled beam spans 49-ft. long. This resulted in an increase in the cost of the project amounting to approximately $180,000.

The total length of the bridge measured c. to c. bearings on the abutments is 7375'-6" as is shown in Fig. 1. It will be noted that the lengths of some of the steel spans are 15" shorter than

the distances c. to c. piers and that some piers appear to be spaced slightly closer than adjacent piers supporting the same type of span. This was done in order to obtain uniformity in steel span lengths for duplication in the fabrication of the superstructure.

The approach grades are 4.75 per cent. The grade lines meet over the center of the Terminal channel with a PVI at Elev. 180.00. The vertical curve between them extends over a length of 1620 ft. between the east and west anchor piers of the Terminal Channel crossing. The roadway grade at the west abutment is at Elev. 31.93. The 4.75 per cent grade continues beyond the abutment. On the east side, the 4.75 per cent grade intersects the level grade on the beam spans at Elev. 22.00 east of Pier 19E which is the junction of the girder and beam spans.

The bridge has two 24-ft. roadways separated by a 4-ft. mall, 5-3/4 in. high. The mall is mountable at slow speed in an emergency. The 10-in. outer curbs have a clear width of 2-ft. thus providing an emergency walk on each side of the bridge.

Except on the 810-ft. main span, the deck consists of a 7-in. reinforced concrete slab on which are superimposed, in separate pours, the center mall and the outer curbs. A longitudinal construction joint extending through the mall and the slab occurs over the continuous line of stringers placed on the center line of bridge to permit a construction joint at that location. This is desirable for operation of a finishing machine for the deck slab of a 4-lane roadway.

On the main channel span, the deck consists of 5-in. deep steel open grating and the mall and outer curbs are formed with raised pattern steel plates.

The railings on the girder and rolled beam spans are made of cast-in-place reinforced concrete. They are low, longitudinal "fence" type railings 2'-6" high. The railings on the truss spans are made of structural steel designed to match the appearance of the concrete rails.

DESIGN CRITERIA

In general, the design is based on the provisions of Division III of 1949 Standard Specifications of the American Association of State Highway Officials. The live load is H20. One exception to the AASHO Specifications pertains to wind loads and the unit stresses used in conjunction with them because those specifications are not considered to be applicable to high-level bridges in locations subject to hurricanes.

Both the substructure and superstructure were investigated and designed for wind loads of 30, 50 and 75 psf intensity. The basis for the design of the superstructure both during erection and in the completed bridge is given in Fig. 2. Of particular significance is the distinction between members designed primarily for D + L + I and those which are designed primarily for wind loads or a small dead load plus wind. Also of significance is the use of yield point or slightly below yield point stresses in conjunction with a 75-psf wind together with the requirement for stability against overturning in a wind of that intensity. The objective of the criteria given in Fig. 2 is to obtain safety under the most severe conditions that can be reasonably anticipated and to obtain greater

uniformity in the margins of strength in the various component parts of the structure, without being extravagant in the use of materials.

For the substructure, basic unit stresses of fc = 700 psi above water, fc = 500 psi below water and fs = 18,000 psi everywhere were adopted for dead plus live load design. These basic stresses were increased for conditions A and B.

For condition C, the piers were checked for stability under a wind load of 75 psf and the unit stress in the steel was limited to a maximum of 30,000 psi.

Steel H-pile foundations were designed on the basis of 50 tons per pile (for 14-in. piles) for dead load plus live load and 65 tons per pile on any exterior row or 70 tons per pile on any corner pile under either condition A or B (Fig. 2). Pile loads were not checked for a wind load of 75 psf since it is considered that such a wind pressure is necessarily a gust pressure and that the inertia of the structure will ordinarily prevent loads of that character from fully loading the piles. Moreover, the piles are believed to be fully capable of sustaining such infrequent and temporary overloads.

These are the same design criteria as those used for the design of the Lower Tampa Bay Bridge.* That bridge is a two-lane bridge having a longer main span. The influence of the design criteria on both the substructure and superstructure was considerably greater on the Tampa Bridge than on the heavier, wider Mathews

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* See Civil Engineering - April 1952.

Bridge.

To those who are familiar with the fact that a hurricane has never within the period of record of the U.S. Weather Bureau struck Jacksonville and that the highest recorded wind velocity is 55 mph, it may seem strange that this bridge was designed for 75 psf wind pressure. However, the paths of many hurricanes surround Jacksonville. The designers were not willing to concede that hurricanes will throughout the life of the bridge continue to by-pass Jacksonville. The only objection was raised by the superstructure contractor who felt that 1952 would not be the year for an exception to occur, but no special guarantees were offered.

A second exception to the AASHO design specifications occurs in the use of minimum thickness of metal in compression members and in the stiffener spacing for plate girders. The AASHO Specifications require a minimum thickness of 1/32 and 1/40 of the distance between the nearest lines of connecting rivets for carbon steel web and cover plates respectively. The Mathews Bridge design follows the Specifications of the American Railway Engineering Association which permit the denominators, 32 or 40, to be multiplied by $\sqrt{p/f}$ when p, the allowable unit stress, is greater than f, the actual stress. The practice of using thinner plates in compression members that are not fully stressed has been adopted by many engineers. Similarly, the AREA Specifications for plate girder stiffeners were adopted in the belief that designs for highway bridges need not be more conservative than those for railway bridges.

FOUNDATION CONDITIONS

The subsurface strata found at the bridge site are more or less typical of those found elsewhere in Florida. The principal bearing material is a calcareous sandy clay substance commonly called marl. At the site, this material extends to a great depth. On top of the marl and under the usual sand and silt overburden, there is usually a rock crust often composed of several separated thin layers but sometimes consisting of a single layer several feet in thickness.

Although the materials encountered are typical with other locations and more or less typical throughout the length of the bridge, there is nothing typical about the arrangement and sequence of the materials from the surface down. Extreme variation occurred not only from pier to pier but within the area of a single pier base. This was generally anticipated from the borings sunk in advance of the design but not all of the extremes in the position and thickness of the overlapping lenses of varying thickness, hardness and stratification could be predicted. There was a wide variation in the lengths of the steel H-piles - even within the same footing - and at one of the large river piers, piles were unexpectedly omitted entirely.

In general, the proper foundation for conditions such as those found at this site is steel H-piles having a specified minimum penetration of 20 ft. below the bottom of the footing. Because of the variation in the lenses of rock overlying the marl and because the marl itself varied from soft to hard (60 blows per ft. on a 2½ in. O.D. sample spoon from a hammer delivering 5400 in.-lbs. per blow), it was difficult to predict

the length of bearing pile that would produce the specified bearing capacity as determined by the resistance to the pile hammer. Sometimes, the problem was to obtain the specified minimum penetration.

All but two piers are supported on piles generally driven as friction piles into the marl because the rock strata are too thin and too unreliable for spread footings because they are usually underlain by marl that is too soft to support, in turn, the rock. Moreover, the rock lenses are seldom uniformly distributed throughout the area of the base and, in some instances, they cover only a portion of the base. Only a few of the piles are supported principally by end bearing on rock rather than by friction.

At some of the piers, unusually heavy spudding and driving were required to penetrate the rock strata. Sometimes the contractor used two driving rigs simultaneously at the same pier. A McKiernan-Terry S-8 hammer drove the heavily reinforced 14-in. W.F. spud in advance of the driving of the permanent bearing piles by an 11-B-2 hammer.

The west approach provided the exception to the general statement that the piles are friction piles driven into marl. On the west shore, sand and silt extend to a depth of 30 ft. below ground level at Elev. 10 and are underlain by lenses of limestone of varying degrees of hardness and thickness having an overall thickness of 20 to 30 ft. Marl occurs below the limestone. At nearly all of those piers, steel H-piles driven into the rock lenses to required bearing capacity and minimum penetration were satisfactory - especially for the smaller

footings supporting the shorter piers.

Conditions at Pier 1W - the west main pier - were particularly annoying. A sound rock stratum about 10 ft. thick exists between Elevations -25 and -35 and this was underlain by firm marl to a satisfactory depth. There was a strong temptation to found the pier on the rock at or near Elev. -27 and utilize the remaining 8 ft. of rock to further spread the superimposed load on the marl. But the present channel is not yet dredged to the full width of the navigation clearance required by the U.S. Engineer Department permit. Not only might the channel be widened to the fender line but it might ultimately be deepened to 40 ft. from its present dredged depth of 32 ft. A pier founded at Elev. -27 on an 8-ft. layer of rock that might be cut off by blasting and undermined to a depth of 5 ft. or more within a horizontal distance of 10 ft. from the edge of the base was not to be considered. Hence, the better foundation material was removed and the pier founded on marl without piles at Elev. -45.

The conditions at the east main pier were entirely different. Here the bottom was so soft that the casing for the borings sunk to Elev. -70 under its own weight and the cofferdam sheet piles instead of being driven were virtually hung on the bracing frames which, in turn, were supported by timber piles. The 450 steel piles supporting this pier were driven to about Elev. -110. Lateral stability was achieved by placing the bottom of the base at Elev. -55 which is 10 ft. lower than normal depth. In addition, the effective bearing area resisting lateral movement was further extended by anchoring the steel sheet piles into the

pier base near the bottom of the seal and near the top of the distribution slab poured in the dry. The sheet piling was burned off 1 ft. above the top of the base and the bottom lengths left in place. Each sheet then acts as a vertical cantilever extending 15 ft. below the base and, because of the anchors spaced 26 ft. on centers, it is effective in resisting lateral motion of the pier in any direction.

East of Pier 1E, the rock lenses rise and the depths of sand and silt become less and, except at Pier 2E, more firm. A decision was made during construction to anchor the sheeting at Pier 2E and leave the lower portion in place as at Pier 1E. At 2E, the anchors were welded to the sheets after the seal had been placed and the cofferdam unwatered. Hence, they engage only the dry slab and are not as efficient as those at 1E where the anchors were welded to alternate sheets before they were driven and where the lower anchors engage the concrete in the base near the bottom. Such anchors are effective when the direction of the lateral force is such that the sheet pile tends to pull away from the base at the bottom.

The rolled beam spans are supported on twenty precast concrete pile trestle bents. All piles are 20 in. square. At the first five bents to the west, the soft silty sand is more than 50-ft. deep. The limestone is thin and the piles, which are 75 to 95-ft. long, are driven into the marl. The five easterly bents toward the shore are located where the sand is more compact, is about 30-ft. deep and is underlain with thicker limestone. The piles drove to the rock but with firm and increasing resistance in penetrating the sand above the rock.

At the ten interior bents, soft silty sand overlies thin layers of limestone which had to be penetrated to develop adequate driving resistance into the marl beneath. This condition required what is referred to as a composite pile - a precast concrete pile with a steel H-pile section embedded in and protruding from the tip. The piles in these bents are about 60-ft. long. Each has a 12-ft. length of 8" HP 36-lb. with 6 ft. of that length embedded in the tip. At one or two bents, the piles driven through soft sandy silt showed a tendency to "walk" on the rock before the tip was seated but aside from that no difficulty was encountered in developing the required resistance for the design load of 45 tons.

In constructing the girder and truss span piers in the river - generally in sequence westward from Pier 19E to Pier 2E - the piles drove without spudding until Pier 8E was reached. From 8E through 3E, the layers of rock became thicker and harder. At Pier 4E where no previous boring had been taken, an unexpected lens of rock underlain by firm marl was found. The heavy spud pile driven by an S-8 hammer could not penetrate the rock. A drill rig was brought in and borings were sunk at the center of the pier and at each corner of the base. Firm rock and marl were found at all locations. The decision was to use a spread footing but the base had to be lowered 4 ft. to reach firm material. Fortunately, the sheet piling was long enough and it was driven down to seat it on the rock. However, an extra bracing frame was required to prevent the sheets from kicking in and also to keep the bending stresses within reasonable limits.

Pier 3E threatened to be one of those cases that become a bridge engineer's nightmare. Pile driving started on the north side of the dam and proceeded southward without unusual difficulty. The outer row of piles was longer than was anticipated from the single boring at the center of the pier. The tips in the outer north row penetrated about 74 ft. below the bottom of the base to about Elev. -110. Each successive row was 2 to 12 ft. shorter until, at the center line of bridge, the piles attained bearing capacity at about Elev. -60. Then a lens of hard rock was encountered which the heavy spud could not penetrate. The spud was tried at several locations in the south half of the dam. No penetration was possible anywhere.

A drill rig was brought in and a few borings disclosed the fact that lenses of rock having an overall thickness of about 10 ft. intruded into the area below the pier base at a depth of 4 ft. below the bottom of the excavation. The thicker lenses did not quite reach the center; hence, they were not disclosed by the original boring at the center of the pier. The overall thickness was composed of several layers of rock varying from 0.3 ft. to 4.5 ft. thick interbedded with soft or very soft clay, marl and limerock, and in two instances with apparent cavities 1.8 ft. and 0.7 ft. thick.

This situation provoked considerable discussion. Some felt that the south half of the base could be founded at a lower elevation as a spread footing bearing on the rock strata about 6 ft. lower than the plan depth of the base. Others were of the opinion that the marl beneath the rock was not sufficiently firm to prevent settlement of the 153-ft. high pier and that settlement

of the south half would not only result in tipping the pier laterally but would result in a completely unknown distribution of stress on the base and no assurance that further tipping would not occur - particularly under the action of lateral loads. The final decision was to use piles throughout the entire base.

At first, it was thought that a large percussion drill would be required for drilling through the rock at each of about 100 pile locations in order to drive the piles. This would, of course, be a costly procedure. However, the contractor suggested that lines of small bore holes be drilled across the dam in succession and that small blasting charges be inserted to break up the rock one row at a time. The free edge near the center made this plan feasible. The rock was sufficiently shattered to permit driving the spud in advance of the permanent pile and the work was successfully completed in accordance with that plan.

SUBSTRUCTURE

The seven piers supporting the truss spans have twin shafts designed as a frame in conjunction with the rigid top strut between them. Because of their location in water 20 to 30 ft. deep and the danger of being struck by ships or barges, the shafts are single and solid below Elev. +20 for Piers 2W, 1W, 1E and 2E, Elev. +15 for Pier 3E and Elev. +10 for Piers 4E and 5E. The solid section projects about 8 ft. at each end beyond the outer faces of the twin shafts above to provide additional protection for the upper part of the pier. The twin shafts have a 2-ft. off-

set in transverse width at mid height.

The approach piers are also twin-shafted piers. Each shaft is supported on a separate footing. The higher piers (3W through 12W and 6E through 8E) have rigid top and bottom struts to provide a four-sided frame to resist transverse loads and are so designed. The bottom strut is omitted at Piers 13W through 17W and 9E through 19E. Piers 18W through 25W have no top or bottom strut; each shaft is designed as a free standing vertical cantilever to withstand 75 per cent of the total transverse force and 50 per cent of the total longitudinal force at the pier. All twin-shafted piers with top struts have a batter of 1/4 in. on 12 in. from top to bottom on three faces, the inside face towards the center line of bridge is vertical. The shafts of the piers without top struts have the same batter on all four faces.

The west abutment is a spill-thru type and is designed as a horizontal beam 118 ft. long supported laterally and vertically by four pilasters. The center pilasters also support the girders in the end approach span. The east abutment has cantilever front and wing walls.

In calculating the depth of tremie seals, an allowance of 3.5 to 5 tons per pile was assumed as effective in resisting hydrostatic uplift when the cofferdam was pumped out to pour the reinforced distribution slab in the dry. Even with this allowance, the largest tremie seal, which occurred at Pier 1E, is 20-ft. thick and required 4096 c.y. of concrete. This was poured in one continuous pour of 40 hours and the contractor's floating concrete plant produced and placed an average of 103 c.y. of concrete per hour. The distribution slab is 12-ft. thick. Some appreciation

of the size of the base may be gained by comparing it to a 3-story building occupying half a city block. The plan dimensions are 54'-6" by 107'-0". Previous reference has been made to the soft bottom of the river at the site of this pier and to the 450 steel H-piles which support the pier.

SUPERSTRUCTURE

Each of the 21 rolled beam spans is of simple beam design. The beams are spaced 6'-6" ctrs. and have a span length of 49'-0". They are detailed with two fixed or two expansion ends of the adjacent spans on each bent. In order to reduce to a minimum the number of expansion dams, they are located at every fourth bent.

The four spans acting as a unit between expansion dams are fixed at the middle to a bent having battered piles; at the quarter points the spans are fixed to bents which rock longitudinally in conformity with temperature changes in length of the stringers. The stringers are 30" WF 130 lb. with 15" channel diaphragms at the middle and ends of each span. At fixed bents 18" WF 50 lb. diaphragms are used to support the transverse construction joints in the concrete deck. The roadway expansion dams consist of cantilever fingers flame-cut from 1-3/4" plates; those in the curbs and mall are formed with bent plates, one sliding over the other.

Each deck girder span consists of two girders spaced 39'-0" ctrs., all girders being 8'-6½" deep b.b. flange angles except for a depth of 7'-6½" for the 78'-9" spans. The cantilever brackets supporting the outer portions of the roadway deck are 9'-4" long and their depth varies from 2'-0" at the outer end to

5'-0½" at the girder. All floorbeams are plate girders 5 ft. deep and have a knee brace at each end. All stringers are 21" WF beams varying from 62 lb. to 73 lb. and spaced 6'-6" apart.

In general, every other pier supports the fixed ends of the adjacent spans and the alternate piers support expansion ends. The piers were designed for longitudinal forces in accordance with that arrangement of expansion joints. The alternate arrangement results in the elimination of half of the expansion joints which is not only economical but it also reduces the maintenance costs which inevitably result from even the best of expansion joint details.

Where expansion joints occur, the ends of adjacent girders are on rocker shoes normally set 2'-6" apart. This distance provides ample space for maintenance between the end floorbeams. The expansion dams are similar to those on the stringer spans.

One feature of the girder spans is the detail of the support at the fixed ends, where the up-grade end of one girder is extended 1'-11" beyond its center of bearing to provide a pin support for the down-grade end of the adjacent girder. (See Fig. 3). As a result, only two fixed shoes support four girders at a fixed pier,and a single floorbeam and its brackets support the stringers in the end panels of both spans. A transverse construction joint is placed in the concrete deck over the common floor beam and brackets.

It will be noted that although the girders are free to rotate under live load, the stringers are somewhat restrained. However, the difference in rotation between the ends of two opposite

stringers is small and angles having 6" outstanding legs are used for the stringer connections to relieve the strain at these connections.

The metal thus saved in comparison with the usual framing with separate floor beams, brackets and shoes amounts to about 8000 lbs. at each pier or a total of 136,000 lbs. for the 17 supports in this structure. The erection of these spans is simpler in proceeding upgrade toward the truss spans. The same detail has been used in a number of other large bridges and the results have been satisfactory.

The trusses in all spans are 60'-0" c. to c. This is sufficient to provide for a deck width of 57'-4" c. to c. railings and avoid a troublesome condition in which the railings have to be framed between the truss members. All panel lengths are 33'-9" except in the anchor and cantilever arms on piers 3E and 4E where they are 33'-10½". All vertical truss members are truly vertical under full dead load and vary from 45 to 105 ft. in height between centers of chords on the main spans and are uniformly 45 ft. on the approach truss spans. Thus, the chords of the latter spans are parallel to the grade.

The in to in of gussets is 26½ in. and the chords are 30½ in. deep b.b. angles. The web members vary from 30 to 15 in. in depth. Cover plates with 9" x 18" perforations at 3 ft. centers are used on the bottoms of all chords and on two opposite faces of the web members. The floor beams are 6'-0½" deep and the stringers are 27" WF 94 lb. beams spaced 6'-6" centers except on the main cantilever span beneath the steel open grating where the floor beams are 4'-10½" deep and the stringers are 24" WF 76 lb. beams spaced

4'-6" ctrs. resting on the top of floor beams. The top laterals are framed as a K-system while the bottom laterals are X-bracing with the center intersections located at alternate floor beams.

The trusses over both channels are of cantilever design, and each suspended span truss has ten panels 33'-9" long. In the main span, the cantilever arms have seven panels and the anchor arms twelve panels. In the approach trusses over piers 3E and 4E the cantilever arm has three panels and the anchor arm eight panels. These proportions were selected with due regard for the use of steel open grating in the main channel span between Piers 1W and 1E and concrete deck on the span over the Arlington Cut between Piers 4E and 5E and to obtain positive reactions at the anchor ends under all combinations of dead, live, and wind loads. As a result, only a few members in the anchor arms undergo reversal of stresses and no embedded anchorages are required at the anchor piers. As an added precaution, steel rings are used to tie the upper and lower shoe castings together at the ends of the pins in order to engage as much dead load as the anchor bolts are capable of supporting.

Metal used in this structure is largely structural carbon steel. However, truss chords having a total stress of 1700 kips or more and web members having a stress of 1200 kips or more are made of structural silicon steel. The cross sectional area of the largest silicon steel member is 172.0 sq.in. and that of the largest carbon steel member 110 sq.in. Shoes in the girder and truss spans and the expansion dams on the truss spans are cast steel.

Erection of the steel superstructure was accomplished by deck travellers. No falsework was necessary for the girder or beam spans but some of the truss spans required falsework bents. Erection of the approach trusses from Piers 5E to 2E was accomplished by continuous cantilever erection using temporary connections between spans and temporary bents where required. Erection of the main spans was symmetrical using falsework bents under the two anchor arms and cantilevering the channel span from both sides to effect a closure at the center.

One feature of the erection which affects the design of structure as a result of erection requirements is worthy of mention. On the anchor arms of the main spans the total shortening in the bottom chord amounts to 1 5/8" under the erection loads just prior to the center closure. At first, the stringers were detailed in these 12 panels to fit the shortened chords in order to facilitate the stringer erection. They would then be permanently short after the suspended span is swing and full dead load is in place. The floor beams which are quite flexible about their vertical axes would probably bend horizontally toward the channel. Sliding supports were therefore added for the stringers on the channel side of the floor beams at the third points of the anchor spans to prevent cumulative distortion of the floor beams.

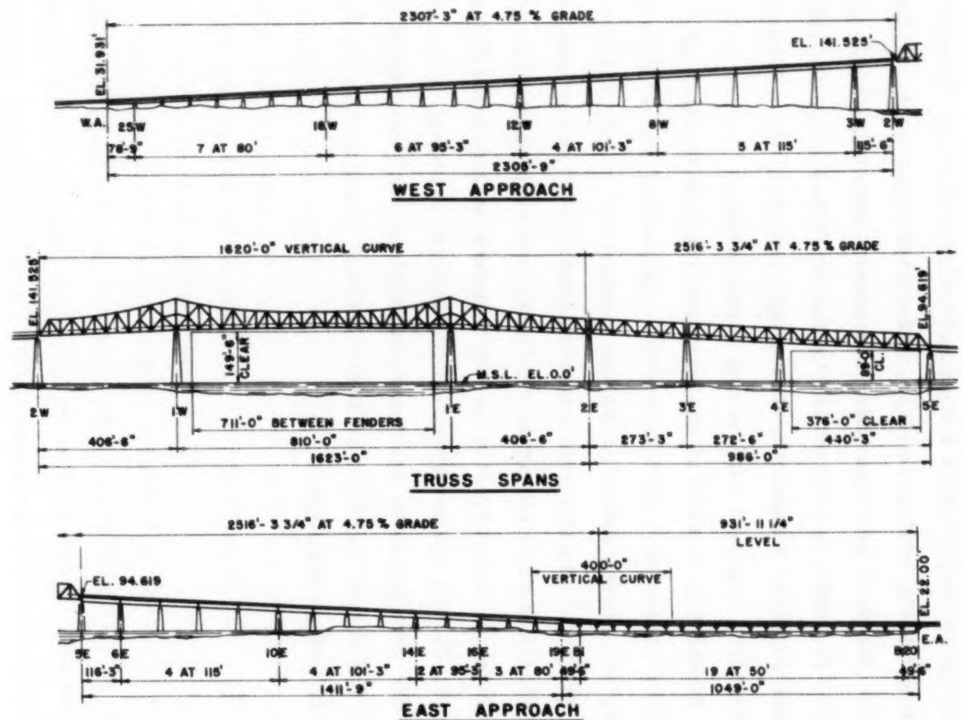
The substructure contains about 62,000 c.y. of concrete and nearly 6500 tons of steel in the form of reinforcing bars, steel bearing piles and steel sheet piling left in place. The superstructure contains about 14,500 tons of structural and reinforcing steel and about 10,500 c.y. of concrete in the deck. The total

construction cost of the bridge abutment to abutment is about $11,450,000.

The bridge was designed and its construction supervised by Parsons,Brinckerhoff,Hall & Macdonald of New York and Reynolds Smith and Hills of Jacksonville, Associated Architects and Engineers. The work was done under the general supervision of Maurice N. Quade, a partner in the former firm. Richard S. M. Lee was Project Engineer on the design and Stanley L. Johnson was Resident Engineer supervising construction.

Except for an initial contract for the construction of 8 approach piers (3W - 10W) by the Geo. D. Auchter Co. of Jacksonville, the entire substructure was constructed by Merritt-Chapman & Scott Corp. The superstructure was fabricated and erected by the Bethlehem Steel Co. with the Industrial Contracting Co. & Associates as subcontractor on the concrete deck and the Miller Electric Co. as subcontractor on the bridge lighting.

The work was done for the Florida State Road Department of which Mr. Sam P. Turnbull is State Highway Engineer and Mr. W. E. Dean is Bridge Engineer.



THE JOHN E. MATHEWS BRIDGE - JACKSONVILLE FLORIDA.

FIG. 1

THE JOHN E. MATHEWS BRIDGE
TABLE OF UNIT STRESSES FOR SUPERSTRUCTURE DESIGN

		LOADING	UNIT STRESS
		FOR MEMBERS DESIGNED PRIMARILY FOR D + L + I :	
		D + L + I	Basic
WIND	A	D+L+I+ W_{30} : 30 p.s.f. on Structure; 200 p.l.f. on Live Load	1.25 X Basic
WIND	B	D + W_{50} : 50 p.s.f. on Structure.	1.40 X Basic
WIND	C	D + W_{75} : 75 p.s.f. on Structure.	See Note "X"
WIND		FOR MEMBERS DESIGNED FOR WIND (OR A SMALL DEAD LOAD + WIND):	
WIND	A	W_{30} or D + W_{30} : 30 p.s.f. on Structure; 200 p.l.f. on Live Load	Basic
WIND	B	W_{50} or D + W_{50} : 50 p.s.f. on Structure	1.25 X Basic
WIND	C	W_{75} or D + W_{75} : 75 p.s.f. on Structure	See Note "X"
ERECTION + WIND		FOR ALL MEMBERS :	
ERECTION + WIND		Erection E : Stress from all Steel Dead Load in place + 50 T Traveller + 200 p.l.f. of bridge for erection equipment	1.25 X Basic
ERECTION + WIND		E + W_{30} : 30 p.s.f. on Structure, Traveller and Equipment	1.30 X Basic
ERECTION + WIND		E + W_{50} : 50 p.s.f. on Structure, Traveller and Equipment	1.50 X Basic
ERECTION + WIND		E + W_{75} : 75 p.s.f. on Structure, Traveller and Equipment	See Note "X"

Note "X"

Stresses in any member shall not exceed the yield point.
Stability of Span as a whole to be investigated for overturning and anchorage to resist uplift to be provided where needed.

FIG. 2

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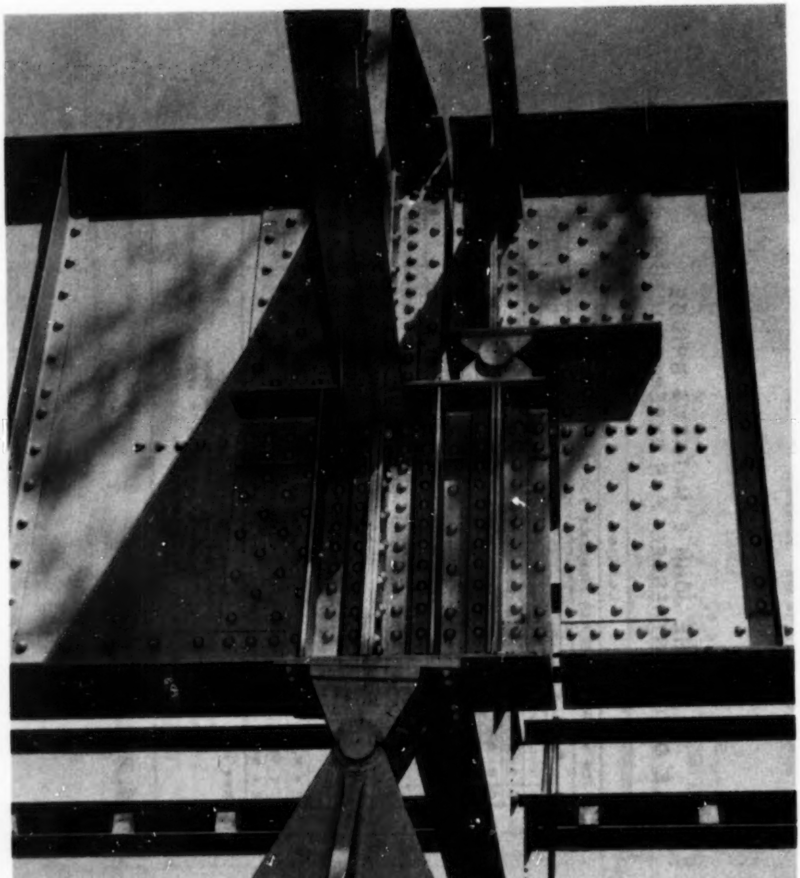

GIRDER SUPPORTS AT FIXED PIERS

FIG. 3

THE JOHN E. MATHEWS BRIDGE
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FIG. 4

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